

A Crack Is Not a Crack: Root Cause Investigation and Monitoring Strategies

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This article, which is the ninth in this series, focuses on root cause investigations, development and deployment of monitoring systems, and interpretation of monitoring data related to concrete cracking. It presents a high-level discussion and is not intended to cover all cases and situations that we may encounter when evaluating concrete bridges for cracking. To facilitate this discussion, let us use two distinctive examples.

Example 1: Large Footing Affected by Alkali Silica Reaction and Delayed Ettringite Formation

Let us consider a case in which a column supported on a footing shows signs of map cracking. As part of the root cause analysis, several cores are taken from the column, and those samples suggest that the damage may result from alkali-silica reaction (ASR). In many routine bridge inspections, investigators do not need to expose a footing and examine its condition. However, in this case, the investigative team considers the background information and findings for the columns and decides to expose the footing for examination. **Figure 1** shows the appearance of the exposed footing.

Because the general appearance of the crack pattern shown in Fig. 1 supports the possibility of ASR in the footing, additional coring is conducted. The petrographic examination of the cores from the footing reveals internal microcracking of concrete attributable to ASR, as well as evidence of delayed ettringite formation (DEF). In other words, the observed cracking is due to ASR and DEF expansions. While most cracking can be attributed to ASR, the

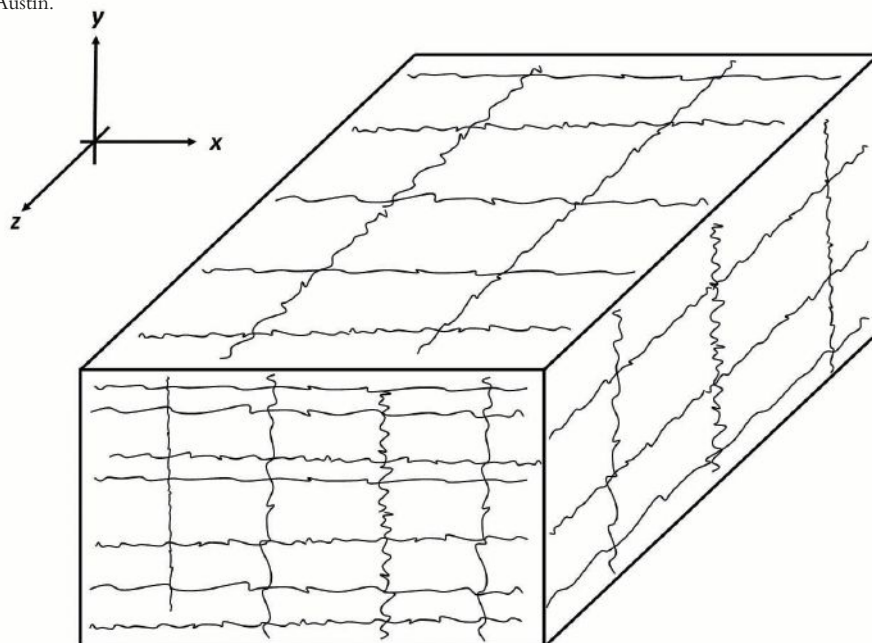
presence of DEF makes the situation a little more complicated because maximum expansion potentials of DEF are typically higher than ASR. (For more information on ASR and DEF, see articles in the Summer 2018 and Spring 2019 issues of *ASPIRE*®.)

With these observations serving as a backdrop, the team decides to use Concrete Works software^{1,2} to further investigate whether the damage was related to DEF. Thermal modeling of the concrete footing was conducted with incomplete information because the time of day at which the footing concrete was placed was unknown. This modeling led the investigative team to conclude that curing temperatures were likely in the 170°F to 175°F range. Because that temperature range is above the DEF-threshold of approximately 158°F, results from the

thermal modeling support the findings of the petrographic examination. In summary, the investigation that started out with map cracking readily observed in a column reveals that the column has ASR-related cracking and the footing supporting the column is affected by both ASR and DEF.

Given the findings from the root cause analysis, instrumentation of the footing with strain gauges to monitor further cracking is warranted (as is instrumentation of the column supported on the footing, but that is not the subject of this discussion). As we proceed, we need to remember the fundamental physics of the degradation mechanisms at play. Because ASR and DEF are bulk expansion mechanisms, it is appropriate to measure expansions in the x, y, and z directions (Fig. 1) and calculate the volumetric expansion due to ASR and

Figure 1. Footing with extensive cracking in all three directions. All Figures: University of Texas at Austin.



DEF. This information helps us determine the structural implications of the observed degradation of the footing.

Another key consideration is the expansions that occurred before the investigation. That is to say, while the instrumentation that we deploy can measure expansions that occur after the strain gauges are placed, we also need to use engineering principles to estimate how far ASR and DEF have progressed to date. To that end, the Damage Rating Index (DRI) may, at first glance, seem to be a useful tool.

DRI is a technique used by petrographers in which specific microscopic damage features are counted or quantified and then multiplied by weighting factors and summed to create a single DRI value. While tracking the DRI values and the change in those DRI values over time may potentially offer benefits, it is not possible at present to reliably convert a DRI value into the amount of expansion (the key indicator that relates to structural implications). Different concrete mixtures may produce different DRI values for the same expansion levels, and exact documentation of this type of information remains incomplete and therefore uncoded. Further, the DRI method is subjective, so DRI values calculated by one petrographer typically do not match the values calculated by another one.

Another technique to estimate the amount of expansion caused by DEF and ASR uses the gauge lengths along the x, y, and z axes. From these lengths, we can assume that expansions are concentrated in crack openings and any straining in the concrete between the cracks is much smaller in magnitude. In this way, the widths of the cracks can be measured by using crack comparators or crack microscopes, and an estimate for the expansions can be obtained. Estimating expansion values by this technique is based on several assumptions, which should be considered when evaluating the determined values. Those assumptions are as follows:

- The contribution of straining in between the cracks is omitted. This is omitted because these deformations are smaller in magnitude in relation to the deformation contribution of crack openings.

- The crack openings are assigned to ASR and DEF expansions only.
- Deformations due to creep- and shrinkage-related volumetric changes are omitted because they are smaller in magnitude in most cases and this omission leads to conservative estimates for the expansions assigned to ASR and DEF.

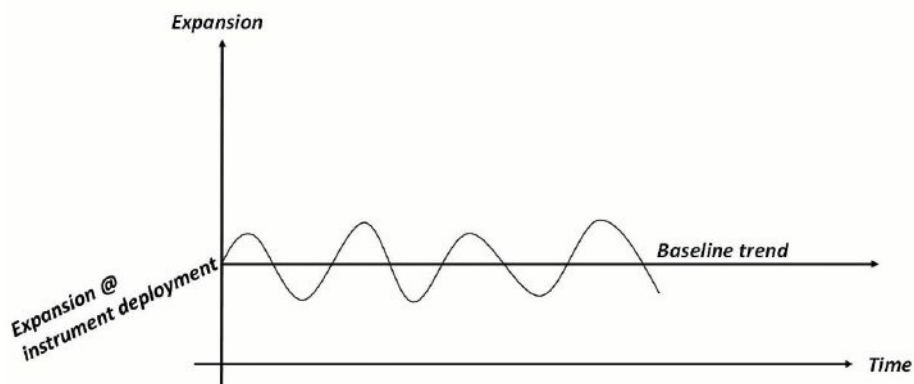
Although these assumptions are common in any engineering analysis on investigations that relate to ASR and DEF, the variables and assumptions involved in a specific investigation should always be explicitly stated in documents that report the findings.

Selection and installation of a reliable deformation monitoring system is also important to measure future expansion. Commonly, surface-mounted targets are used and relative movement of those targets is measured. The durability of the target installation is important because to trend data, measurements must, at a minimum, span over a period of 2 to 3 years. To increase durability, it is advisable to drill holes that reach into the structural core (that is, the holes penetrate behind the reinforcement near the exposed concrete face) and install stainless steel pins using an epoxy adhesive. Measuring from stainless steel pins more accurately represents the expansions experienced by the structural core than those measured from surface-mounted targets. Once the stainless steel pins are installed, initial readings can be taken in the x, y, and z directions and the datum of expansions in each of these three directions can be set. Importantly, future expansions can be measured by using these pins and their relative movements against each other.

Let us next discuss possible trends that may be observed in the data. By focusing our attention on the expansion readings taken in the x, y, and z directions, we can see an expansion-time history similar to that shown in Fig. 2.

In Fig. 2, the vertical line offset seen in the expansion axis is determined using the expansion-to-date estimation technique described previously. The uncertainty of this estimation is repeated here to emphasize that this uncertainty must be included in the subsequent engineering analysis in a conservative manner. Once this datum is established, the almost sinusoidal trend depicted in Fig. 2 can be examined. The expansion behavior shown in Fig. 2, which is one of the cases we will examine herein, is idealized (that is, near sinusoidal) to simplify the subsequent discussion. In actuality, the expansion behavior may be more “jagged” as it may be influenced by daily live-load application on the superstructure, daily temperature variations, and other factors. This subtlety notwithstanding, the overall behavior shown in Fig. 2 is largely influenced by seasonal temperatures, and the baseline trend in this figure seems to be constant. This baseline trend implies that the cracks depicted in Fig. 1 are opening and closing as the seasonal temperature is fluctuating. From these observations, it can be interpreted that both ASR- and DEF-related expansions are influenced by daily live-load application on the superstructure, daily temperature variations, and the exposure conditions that are present on site. In this case, the maximum expansion measurement can be taken from the monitoring systems and converted into structural implications of that expansion

Figure 2. Expansion in the x, y, or z directions shown in Fig. 1, case 1.



level by using guidance, such as those presented in the *fib* (International Federation for Structural Concrete) Model Code 2020.³ This evaluation can be done in all applicable directions; most importantly, it can be done in the direction and behavioral mode that controlled the design in the first place. For example, if shear strength of the footing was the controlling behavioral mode in the original design, we would be most interested in evaluating that mode. If bending about one of the axes of interest controlled the design, that would be our starting point/primary focus of interest.

Let us now consider another possibility in which the baseline trend is not as benign as the trend shown in Fig. 2. **Figure 3** depicts a case in which the seasonal temperature variations result in expansions and contractions that cycle around a baseline trend that shows a net positive expansion. This type of behavior indicates that ASR-related expansions, DEF-related expansions, or both ASR- and DEF-related expansions are still active. The volumetric (bulk) expansions that result from one or both of these chemical degradation mechanisms are ongoing. In other words, reactants that contribute to the reaction mechanisms have not been depleted, and the environmental exposure conditions are still conducive to promoting these reactions. In such a case, interim structural evaluations are desired for the maximum expansion levels documented. Alternatively, a structural evaluation can be made for expansion levels higher than those measured and may be used as limiting expansion levels for the structural evaluations. If the future expansion trends show the expansion has stopped and the trends look like alternative A in Fig. 3, the structural evaluations based on the expansion trends can be finalized. Provided that the maximum expansion levels support structural capacities that are larger than the demands imposed by the governing load cases, we can conclude that there is no structural load-carrying deficiency at play. In the case of alternative B in Fig. 3, the footing is still undergoing expansion, and continued monitoring and structural evaluation are essential. This type of trend does not mean the structure is unsafe. If the maximum expansion levels indicate a

structural capacity degradation that is large enough to create a challenge with load effects, structural retrofits and/or replacement may be needed.

Example 2: Three-Column Bent with Shear and Flexural Cracks

Let us next focus our attention on a three-column bent that is exhibiting a substantial amount of shear distress (Fig. 4). As a starting point, investigators visually inspect the bent to document the widths of the cracks and map the locations and lengths of the cracks so that subsequent inspections can benefit from this datum. The plan set available for this bent indicates that the crack-control reinforcement in the cap is 0.2% in the x and y directions (Fig. 4). Let us also assume the maximum crack width measured is 0.03 in. Using Table 1 from the Summer 2025 *ASPIRE* article in this Concrete Bridge Technology series, we can determine that the cap has been loaded to

30% to 50% of its capacity. This initial assessment is helpful in decision-making.

In addition to evaluating the margin to the capacity of the cap, the investigators in this case conduct a petrographic examination of three cores taken from the cap. This examination reveals no deleterious chemical degradation mechanisms. However, it points to a higher water-cement ratio of 0.55 than the maximum water-cement ratio of 0.45 specified in the construction documents. The higher-than-specified water-cement ratio will result in a greater drying shrinkage tendency. That tendency is a contributing factor to the observed cracking, but it does not fully explain the formation of shear cracks.

To further investigate the cracked condition of the cap, let us consult Section 5 provisions of the American Association of State Highway and Transportation

Figure 3. Expansion in x, y, or z directions in Fig. 1, case 2.

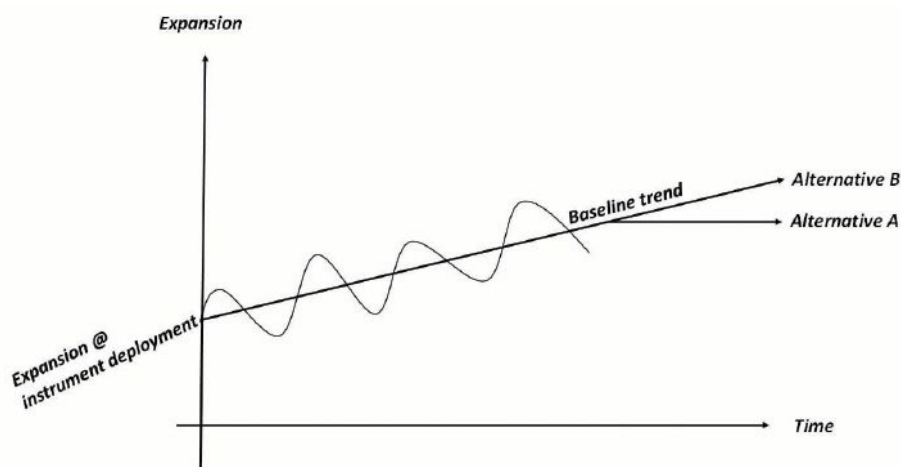
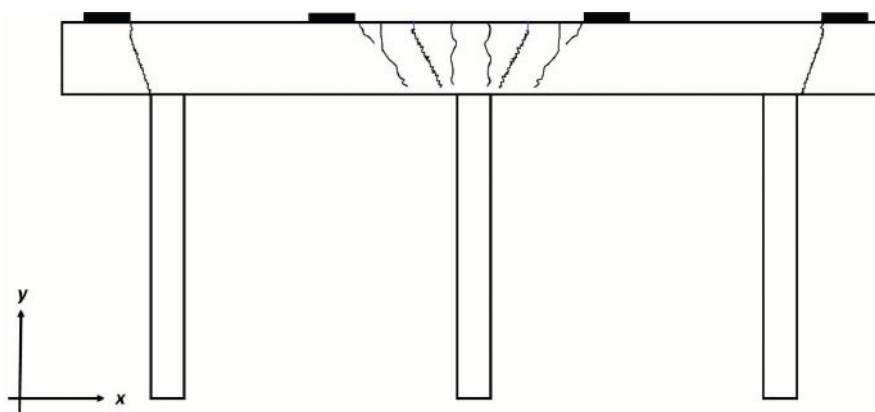


Figure 4. Three-column bent with shear cracking.



Officials' AASHTO LRFD Bridge Design Specifications.⁴ Commentary to Article 5.8.2.2 states the following:

The estimated resistance at which diagonal cracks begin to form, V_{cr} , is determined for the initial geometries of the D-Regions using the following expression (Birrcher et al., 2009^[5]):

$$V_{cr} = \left[0.2 - 0.1 \left(\frac{a}{d} \right) \right] \sqrt{f'_c} b_w d \quad (C5.8.2.2-1)$$

but not greater than $0.158 \sqrt{f'_c} b_w d$,

nor less than $0.0632 \sqrt{f'_c} b_w d$.

where:

a = shear span (in.)

d = effective depth of the member (in.)

f'_c = compressive strength of concrete for use in design (ksi)

b_w = width of member's web (in.)

Where the shear in service is less than V_{cr} , reasonable assurance is provided that shear cracks are unlikely to form.

By using Eq. C5.8.2.2-1 in our example 2, we can understand that shear cracking is expected for the three-column bent shown in Fig. 4. This observation is important in completing the root cause analysis. It is certainly possible to perform a more detailed analysis, consistent with the Levels of Approximation approach articulated in the *fib* Model Code 2020. However, the additional computational effort does not seem warranted in this case because we can explain the observed shear cracking readily, as explained earlier. In addition, restrained shrinkage effects that introduce axial tension forces in the bent cap further exacerbate the cracking potential.

With the apparent margin to failure loads, a minimal amount of instrumentation is needed to monitor the cracks. In this case, inspectors attach targets to a few representative cracks to measure the movement of those cracks. Instruments at three of the widest shear

crack locations will measure the stability of those cracks over time. In addition, the vertical (flexural cracks) over the interior column are also instrumented to complete the instrumentation plan.

Let us next examine a few potential data trends, to complete the picture. Figure 5 shows a potential crack-width time history. In that figure, the widths of the cracks monitored exhibit an opening and closing pattern due to temperature changes experienced by the bent cap. In this case, we have "active" cracks, but they are not "progressive" cracks. They are not progressive because the baseline trend shown in Fig. 5 is horizontal. This condition is quite stable. Therefore, in this case, a justifiable strategy is to perform a durability repair (for example, seal the cracks) and continue the crack-width monitoring for 2 to 3 years. After that monitoring period ends, the original inspection routine, consistent with the owner's inspection program, can be resumed.

Next, let us study the case in which the baseline trend is not horizontal and

the widths of the cracks that are being monitored are increasing over time. This condition signals cracks that are both active and progressive. The progression of cracking should be examined carefully. Specifically, applied loading on the structure, the condition of the foundations, and other potential significant contributors to cracking should be considered.

As the crack widths increase, two scenarios are possible. In the best-case scenario (alternative A in Fig. 6), the baseline trend stabilizes. That is to say, after a few years of progression, the behavior can be deemed stable. In the other alternative (alternative B in Fig. 6), the progression continues and the baseline trend is not stable. In this case, structural retrofit options alongside the replacement option should be evaluated, with due consideration given to safety, durability, and cost. The use of advanced tools, such as the one described by Zaborac et al.,⁶ may also be appropriate.

Conclusion

Although the examples discussed in this article do not cover all of

Figure 5. Shear crack width versus time: case 1.

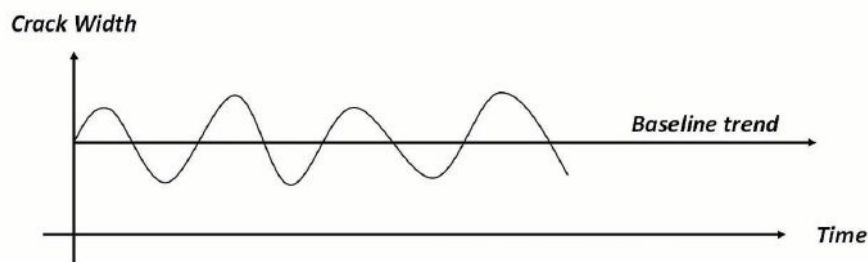
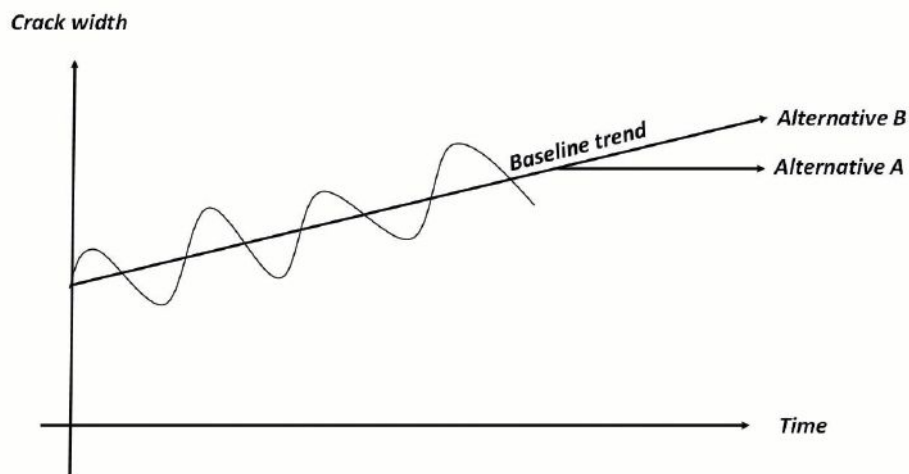


Figure 6. Shear crack width versus time: case 2.



the cases we may encounter, they are notable examples of physical phenomena that can cause distress in structural components. Both examples highlight the need to identify the root cause of cracking before developing any instrumentation or structural monitoring system. Engineers must clearly understand the fundamentals at play before they intervene. In some cases, structural evaluation is reasonably straightforward because there is sufficient guidance available in the bridge industry's design and structural evaluation specifications. In other cases, in-depth linear and nonlinear finite element analyses are necessary.

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